COPY

Materials submitted by Daniel A. Vellone, 51 Church Street, at the Grafton Planning Board meeting of June 24, 2019, during the continued public hearing for "Brigati Village," 41 Church and 14 West Streets (SP 2019-2/SPA):

1) Excerpt from: "Technical Release 210-60, Earth Dams and Reservoirs, March 2018," Conservation Engineering Division, Natural Resources Conservation Service, United States Department of Agriculture, 8 pages.
2) Excerpt from: "Slope Stability," EM 1110-2-1902, 31 Oct 2003, Engineer Manual, Engineering and Design, U.S. Army Corps of Engineers, 4 pages.
3) Excerpt from: "Geotechnical Engineering Circular No. 3, Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I-Design Principles," Publication No. FHWA-SA-97-076, May 1997, Office of Engineering, Office of Technology Applicants, Federal Highway Administration, U.S. Department of Transportation, 2 pages.
4) Excerpt from: "Stability and Performance of Slopes and Embankments – II," edited by Raymond B. Seed and Ross W. Boulanger. ASCE, 1992, 7 pages.
5) Excerpt from: "Soil Mechanics," Design Manual 7.01, Revalidated by Change 1 September 1986, Naval Facilities Engineering Command, United States Navy, 7 pages.

EXHIBIT 43

USDA
United States Department of Agriculture

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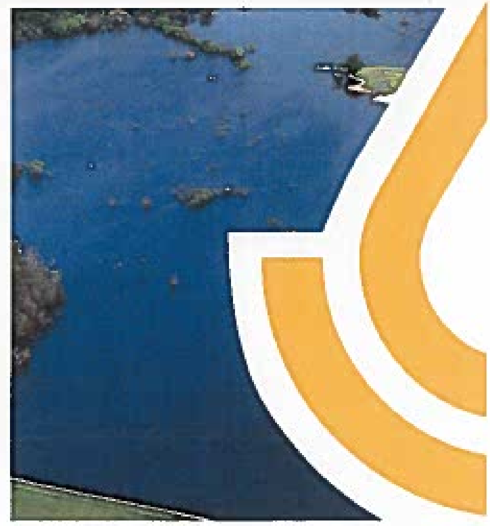
Technical Release 210-60

Earth Dams and Reservoirs

March 2019



Conservation Engineering Division

Natural
Resources
Conservation
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Overall height of embankment, H, (ft)	Minimum top width (ft)		
	All dams	Single-purpose floodwater retarding	Multipurpose or other purposes
35<94.9	N/A	14	(H+35)/5
95.0+	N/A	16	26

Embankment Slope Stability

Analyze the stability of embankment slopes using generally accepted methods based on sound engineering principles. Document all aspects of the analyses performed. Documentation should include analysis program and methods used, input data, and tabular and graphical output. Document shear strength parameters used for each zone of the embankment and each soil type or horizon in the foundation and the basis for these parameters. Document piezometric assumptions included in the analyses and any other variables that will influence stability, along with the basis for the values. The stability analysis should incorporate zones of saturation and seepage pressures consistent with the results of the seepage analysis. Identify and incorporate features necessary to achieve required safety factors in the design.

The extent and intent of the site investigation, soil testing program, and analyses should be consistent. The level of investigation, testing, and analysis should be commensurate with the complexity of the site and structure, along with the consequences of failure. Minimum required static stability safety factors are summarized in figure 5–3 for each condition analyzed.

This TR does not address all potential situations that may arise in the embankment dam stability analysis. When using approaches outside the scope of this TR, document references for all analysis methods used. Include justification for the methodology used, the basis for material characterization and pore pressure assumptions, and comparison of the analysis results with analysis methods included in this technical release.

Consider the following guidance in analyzing slope stability for all conditions:

- Evaluate the overall stability of the embankment and foundation. Determine the stability of critical slip zones (zones that encompass a significant portion of the embankment, pass through major portions of the embankment and foundation, and zones that include the crest), and identify slip surfaces with the overall minimum factors of safety. Document the location of slip surfaces evaluated relative to the location of comparatively weaker materials and areas of high pore pressure, to assure the evaluation of zones that have a propensity to control stability.

- Use effective stress parameters for soils that will drain as rapidly as load is applied. These parameters apply for all conditions of stability analyzed for these soils.

- Limit the use of infinite slope equations and associated safety factors listed in figure 5–3 to the stability of near surface failure surfaces in the exterior slope of embankments with cohesionless soils, when the critical failure is wholly within soils of this character, and analysis of deeper failures would result in higher factors of safety. Infinite slope equations should model the predicted seepage pattern in the slope under analysis. Different equations apply for no seepage, horizontal seepage paths, and seepage paths parallel to the slope face. If shallow slope failure could progress and affect deeper embankment zones, use the higher safety factors listed in figure 5–3. If cohesionless zones occur with other soil types in analysis of a cross section, use other methods to analyze circular arc or wedge-shaped surfaces that intersect the soil zones with cohesion to locate the minimum safety factor.

- Use residual effective stress parameters for modeling slope stability analyses involving fissured clays or shales if preexisting movements have occurred. Drained shear strength tests provide the basis for these parameters. Use residual parameters for designing against shallow slope failures in desiccated clay embankments with previous movement. Designers can use fully softened shear strength with conservative pore pressure assumptions to evaluate the potential for first time shallow slope failures in compacted high plasticity clay embankments.

Static Stability

Analyze embankment stability for each of the following conditions in the structure life that are appropriate to the site. Clearly document the reasons for not analyzing a condition. Document the source of all shear strength parameters used. When using a shear strength parameter determined from an empirical correlation, include correlations to field performance and document the justification for use of the empirically based value rather than site-specific sampling and testing. Include discussion on the sensitivity of the outcome of the analysis to the variability of the parameter. Stability analyses must identify and incorporate any lower strength materials that may control stability.

Stability During Construction

The construction sequence may control the configuration of the embankment and rate of construction when analysis predicts that embankment soil, foundation soils, or both will develop significant pore pressures during embankment construction. Evaluate site conditions, such as soft clay layers in the foundation that may require special designs. Consider flattened slopes and sequentially staged construction to achieve stable conditions. Factors determining the likelihood of this occurring include the overall height of the planned embankment, the speed of construction, the saturated consistency of foundation soils, and others. Perform appropriate shear tests to model placement conditions of embankment soils, as summarized in figure 5–3. Consider the highest likely placement water content of embankment soils in the shear-testing program. Alternatively, designers can use effective stress shear parameters to establish limits for pore pressure buildup during construction. The design should include procedures for monitoring pore pressures during construction to assure that actual pore pressures do not exceed limits established during design.

Construction stability analysis must include stability of excavation slopes that could have significant safety or construction consequences, such as excavation of existing embankment slopes during rehabilitation construction.

Rapid Drawdown

Analyze the stability of the upstream embankment slope for the condition created by a rapid drawdown of the water level in the reservoir from the normal full reservoir level from which a phreatic line is likely to develop to the elevation of the lowest gated outlet. If the potential exists for saturation of zones in the upstream slope during temporary flood retention storage above the permanent normal pool, consider the effect of drawdown back to the normal pool on that saturated portion of the embankment. Designers may use transient seepage analysis to estimate the short-term saturation of the portion of the embankment above permanent normal pool. Select shear strength parameters for the

Develop a phreatic surface resulting from the maximum reservoir elevation. Consider material properties and the potential for defects in the upper portion of the embankment when selecting the appropriate seepage analysis and evaluating seepage conditions that could develop. Also consider the potential for increase in pore pressures in the normally saturated portion of the foundation or embankment that may result from the higher reservoir loading. Evaluate a range of potential seepage conditions to determine the sensitivity of flood storage stability to potential seepage conditions.

If short term flood storage stability is highly dependent on the success of internal drainage features to control the phreatic surface, check stability assuming partially functioning or plugged drainage systems.

For appropriate slopes, designers can use infinite slope equations and applicable safety factors listed in figure 5-3.

Figure 5–3: Table of Static Slope Stability Criteria

Design condition	Primary assumption	Remarks	Applicable shear strength parameters	Minimum safety factor
1. Construction Stability (upstream or downstream slope)	Zones of the embankment or layers of the foundation expected to develop significant pore pressures during construction	Low-permeability embankment soils should be tested at water contents that are as wet as likely during construction (usually wet of optimum). Saturated low permeable foundation soils not expected to consolidate fully during construction. Existing dams with additional fill placed above saturated low-permeability zones.	Unconsolidated; Total stress consistent with preconstruction stress state	1.4 for failure surfaces extending into foundation layers 1.3 for embankments on stronger foundations where the failure surface is located entirely in the embankment
	Embankment zones and/or strata not expected to develop significant pore pressures during construction	Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain as rapidly as they are loaded	Effective stress	
2. Rapid drawdown (upstream slope) (short-term)	Drawdown from the highest normal pool to the lowest gated outlet	Consider failure surfaces both within the embankment and extending into the foundation Low-permeability embankment and foundation soils that will have limited drainage during reservoir drawdown Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain as the reservoir is drawn down	Lowest of effective stress or consolidated; total stress; consistent with predrawdown consolidation stresses (See fig. 5-1) Effective stress	1.2 1.1 for near surface (infinite slope) failure surfaces in cohesionless soils
3. Steady seepage (Long-Term)	Reservoir water surface at highest normal pool. Phreatic surface developed from the highest normal pool; typically the principal spillway crest	Consider failure surfaces within both the embankment and extending into the foundation. Foundation analysis may require separate phreatic surface evaluation, particularly in sites with confined seepage that results in uplift at the downstream toe.	Effective stress (See Fig. 5-4)	1.5 1.3 for near surface (infinite slope) failure surfaces in cohesionless soils
4. Flood surcharge	Reservoir at freeboard hydrograph level. Steady seepage phreatic surface incorporating increased pore water pressure that may occur from flood detention and pore water pressure from short term seepage resulting from reservoir surface above the normal pool elevation	Consider failure surfaces within both the embankment and extending into the foundation. Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain rapidly with changes in reservoir elevation Low-permeability embankment and foundation soils that will have limited drainage as the increased reservoir load is applied	Effective stress Lowest of effective stress or consolidated; total stress (See fig. 5-1)	1.4 1.2 for near surface (infinite slope) failure surfaces in cohesionless soils

Figure 5–4: Shear Strength Parameters, Associated Tests, and Nomenclature

Shear strength parameter	Types of shear tests	NRCS notation	Other common abbreviation	Notes	American Standard for Testing Materials (ASTM) test standard
Total stress; unconsolidated[1]	Unconsolidated undrained triaxial	UU	Q		D2850
	Unconfined-compression	qu		Suitable for limited types of soils[2]	D2166
	Field vane shear			Limited to saturated low strength (Su<2 tsf) clays and silts. Field vane shear test results must be corrected for plasticity index to determine undrained strength	D2573
Total stress; consolidated[1]	Consolidated undrained triaxial	CU	R	ASTM standard for cohesive soils	D4767
Effective stress	Consolidated undrained triaxial with pore pressure measurements	$\overline{CU}$	R	Determine effective stress parameters by subtracting measured pore pressures from total stresses.	D4767
	Consolidated drained triaxial[3]	CD	S		D7181
	Direct shear			Strain rate must be limited to the extent required to allow drainage	D3080
Residual effective stress	Ring shear			Residual shear strength of cohesive materials containing preexisting shear surfaces	D6467
				Fully softened shear strength of cohesive materials without preexisting shear surfaces.	D7608
	Repeated direct shear			No ASTM standard for soils	

1. Using total stress parameters in the analyses model generates pore pressures (positive or negative) in laboratory specimens that simulate the pore pressures that will develop in the field soils during loading. Total stress parameters used in developing composite shear strength envelopes should not consider large negative pore pressures that may result from shear testing. Take the stress condition at failure to correspond to either maximum deviator stress or maximum principal effective stress ratio to define total stress parameters. Only use the stress conditions at a target axial strain to define failure when more limiting than other criteria.
2. The test for unconfined compressive strength (qu) applies only to cohesive materials that will retain strength without confining pressure and will not expel water during testing. This test does not apply to dry, crumbly, fissured, or non-cohesive soils or other soils that will not hold their shape without confinement. Some soils will exhibit higher strengths when tested using the unconsolidated undrained (UU) test. See ASTM D2166.
3. NRCS generally performs consolidated undrained with pore pressure measurements ($\overline{CU}$) tests rather than consolidated drained (CD) tests, as a more rapid strain rate with the CU test reduces the time required to complete the test. With appropriately performed tests, effective shear strength parameters obtained from CU *bar* tests should be equivalent to those from CD tests.

Dynamic Stability (Pseudo-static)

Evaluate the effects of earthquake loading for all dams.

Analysis of dams for earthquake loading must address all potential failure modes. Distress of embankment dams from earthquake loading primarily manifests itself in deformation causing crest settlement, lateral spreading, cracking and differential displacements between the embankment or foundation and appurtenant structures.

Limit embankment and foundation deformation resulting from the design earthquake to an amount that will not result in embankment overtopping from crest settlement, internal erosion from cracking or damage to appurtenances to the extent that could result in dam failure. Dam failure is instability or major damage that may lead to uncontrolled loss of the reservoir when subjected to the design earthquake loading.

Limit damage from the operating basis earthquake to the extent that, after the earthquake and prior to repair, the dam and appurtenances will pass the principal spillway flood without failure.

Design appurtenant structures such that earthquake-caused damage to structures will not lead to dam failure. Evaluation of appurtenances must consider damage that could result from embankment or foundation permanent deformation, in addition to transient seismic forces.

Evaluate the potential for fault movement in the dam foundation and the stability of the reservoir rim.

Do not locate new dams on active faults. Do not locate new significant or high hazard potential dams on capable faults, without specific design features that address potential fault movement.

Existing dams located on active faults must be able to withstand the potential fault offset that could result from the design earthquake, without failure.

Existing dams with high consequence from a seismic failure, located on capable faults, must withstand the potential fault offset resulting from the design earthquake, without failure.

Existing dams located on active faults must withstand the potential fault offset resulting from the operating basis earthquake and must, prior to repair following an earthquake, pass the principal spillway flood without failure.

Design and construct or rehabilitate embankment dams to resist earthquake loading with sound defensive measures representative of the current practice and commensurate with the seismicity of the site, site geology, and hazard of the structure.

Analysis

Seismic analyses should begin with the simplest and most conservative method to analyze the embankment and foundation. If these analyses show that the structure will perform adequately under the design earthquakes, then further analyses are unnecessary. If further analyses are needed, the analyses must be progressively more detailed and complex. Provide a determination of earthquake loading, site characterization and determination of material properties consistent and appropriate for the level of detail and complexity of the analyses.

Sites where the design peak horizontal ground acceleration (PGA) determined in accordance with part 4 of this TR is equal to or less than 0.07 g require no seismic analysis. This low level of earthquake loading should not significantly distress embankment dams, even with site conditions susceptible to damage from earthquake loading.

If the design ground motion exceeds 0.07 g, evaluate the potential for loss of shear strength due to liquefaction or cyclic failure under seismic loading.

For seismic analyses, assume that the reservoir is at the highest normal pool elevation. Base the extent

Collapsible materials are often associated with deposits, such as alluvial fans, terraces, and aeolian soils. If the potential for collapsible soils exists, perform extensive site investigations and testing to provide quantitative information for design and construction. Obtain and test undisturbed samples that are representative of the collapsible material.

Liquefaction Susceptibility

Soil liquefaction typically occurs in recent deposits of loose sand and silty sand located below the water table; however, gravels and low plasticity silts may also liquefy. Assess groundwater conditions and the occurrence and extent of potentially liquefiable soils that could lose strength under the earthquake shaking considered at a site. Characterize other soils that could undergo loss of strength resulting from earthquake shaking, including sensitive clays, clays and plastic silts potentially susceptible to cyclic softening, and collapsible weakly cemented soils. When developing a subsurface investigation plan, the geologist and geotechnical engineer must consider identifying and locating all layers of potentially liquefiable soils to provide data adequate to analyze the potential significant loss of strength under earthquake loading.

Seismicity and Earthquake Loading

Consider the effects of earthquake loading for all dams. The level of investigation and analysis performed for site characterization and delineation of potential earthquake loading will depend on the potential consequences of seismic dam failure, the seismicity of the site, individual site characteristics that influence the performance of the dam under earthquake loading and the anticipated analysis and design methods.

Geologic investigations must document the existence or absence of active and capable faults at the site. Geologic investigation reports must include a map showing all magnitude 4 or intensity V (near epicenter), or greater earthquakes of record and any active or capable faults within a 100-kilometer (62 mile) radius of the site. Use moment magnitude (Mw) to define earthquake magnitude occurring after the 1970s.

Assess the existence and extent of potentially liquefiable soils and groundwater conditions at sites where the considered earthquake ground motions could cause liquefaction in susceptible materials.

As a minimum, use the peak ground acceleration (PGA) resulting from the maximum design earthquake loading (MDE) corresponding to the annual exceedance probability listed in figure 4–1 below. Obtain PGA and other earthquake characteristics associated with the exceedance probabilities listed in figure 4–1 from the U.S. Geological Survey (USGS), Earthquake Hazards Program. Ground motion values must be representative of the site foundation conditions. If foundation conditions representative of the site are not directly available from the USGS data, adjust the PGA values to represent the actual site conditions.

Figure 4–1: Table of Earthquake Loading – Minimum Peak Ground Acceleration for Seismic Evaluation of Dams and Appurtenances

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ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL

Chapter 3
Design Criteria

3-1. General

a. Applicability. This chapter provides guidance for analysis conditions and factors of safety for the design of slopes. Required factors of safety for embankment dams are based on design practice developed and successfully employed by the USACE over several decades. It is imperative that all phases of design be carried out in accord with established USACE methods and procedures to ensure results consistent with successful past practice.

(1) Because of the large number of existing USACE dams and the fact that somewhat different considerations must be applied to existing dams as opposed to new construction, appropriate stability conditions and factors of safety for the analysis of existing dam slopes are discussed as well.

(2) The analysis procedures recommended in this manual are also appropriate for analysis and design of slopes other than earth and rock-fill dams. Guidance is provided for appropriate factors of safety for slopes of other types of embankments, excavated slopes, and natural slopes.

b. Factor of safety guidance. Appropriate factors of safety are required to ensure adequate performance of slopes throughout their design lives. Two of the most important considerations that determine appropriate magnitudes for factor of safety are uncertainties in the conditions being analyzed, including shear strengths and consequences of failure or unacceptable performance.

(1) What is considered an acceptable factor of safety should reflect the differences between new slopes, where stability must be forecast, and existing slopes, where information regarding past slope performance is available. A history free of signs of slope movements provides firm evidence that a slope has been stable under the conditions it has experienced. Conversely, signs of significant movement indicate marginally stable or unstable conditions. In either case, the degree of uncertainty regarding shear strength and piezometric levels can be reduced through back analysis. Therefore, values of factors of safety that are lower than those required for new slopes can often be justified for existing slopes.

(2) Historically, geotechnical engineers have relied upon judgment, precedent, experience, and regulations to select suitable factors of safety for slopes. Reliability analyses can provide important insight into the effects of uncertainties on the results of stability analyses and appropriate factors of safety. However, for design and construction of earth and rock-fill dams, required factors of safety continue to be based on experience. Factors of safety for various types of slopes and analysis conditions are summarized in Table 3-1. These are minimum required factors of safety for new embankment dams. They are advisory for existing dams and other types of slopes.

c. Shear strengths. Shear strengths of fill materials for new construction should be based on tests performed on laboratory compacted specimens. The specimens should be compacted at the highest water content and the lowest density consistent with specifications. Shear strengths of existing fills should be based on the laboratory tests performed for the original design studies if they appear to be reliable, on laboratory tests performed on undisturbed specimens retrieved from the fill, and/or on the results of in situ tests performed in the fill. Shear strengths of natural materials should be based on the results of tests performed on undisturbed specimens, or on the results of in situ tests. Principles of shear strength characterization are summarized in Appendix D.

Table 3-1
Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.

[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.

[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.

[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.

[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.

For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

(1) During construction of embankments, materials should be examined to ensure that they are consistent with the materials on which the design was based. Records of compaction, moisture, and density for fill materials should be compared with the compaction conditions on which the undrained shear strengths used in stability analyses were based.

(2) Particular attention should be given to determining if field compaction moisture contents of cohesive materials are significantly higher or dry unit weights are significantly lower than values on which design strengths were based. If so, undrained (UU, Q) shear strengths may be lower than the values used for design, and end-of-construction stability should be reevaluated. Undisturbed samples of cohesive materials should be taken during construction and unconsolidated-undrained (UU, Q) tests should be performed to verify end-of-construction stability.

d. Pore water pressure. Seepage analyses (flow nets or numerical analyses) should be performed to estimate pore water pressures for use in long-term stability computations. During operation of the reservoir, especially during initial filling and as each new record pool is experienced, an appropriate monitoring and evaluation program must be carried out. This is imperative to identify unexpected seepage conditions, abnormally high piezometric levels, and unexpected deformations or rates of deformations. As the reservoir is brought up and as higher pools are experienced, trends of piezometric levels versus reservoir stage can be used to project piezometric levels for maximum storage and maximum surcharge pool levels. This allows comparison of anticipated actual performance to the piezometric levels assumed during original design studies and analysis. These projections provide a firm basis to assess the stability of the downstream slope of the dam for future maximum loading conditions. If this process indicates that pore water pressures will be higher than those used in design stability analyses, additional analyses should be performed to verify long-term stability.

e. Loads on slopes. Loads imposed on slopes, such as those resulting from structures, vehicles, stored materials, etc. should be accounted for in stability analyses.

condition may be possible if sufficiently detailed studies are made for design, if construction delays are unlikely, and if the observational method is used to confirm the design in the field. Such a condition, where the long-term condition is unstable, is inherently dangerous and should only be allowed where careful studies are done, where the benefits justify the risk of instability, and where failures are not life-threatening.

(3) Instability of excavated slopes is often related to high internal water pressures associated with wet weather periods. It is appropriate to analyze such conditions as long-term steady-state seepage conditions, using drained strengths and the highest probable position of the piezometric surface within the slope. For submerged and partially submerged slopes, the possibility of low water events and rapid drawdown should be considered.

e. *Natural slopes*. The analysis procedures in this manual are applicable to natural slopes, including valley slopes and natural river banks. They are also applicable to back-analysis of landslides in soil and soft rock for the purpose of evaluating shear strengths and/or piezometric levels, and analysis of landslide stabilization measures.

(1) Instability of natural slopes is often related to high internal water pressures associated with wet weather periods. It is appropriate to analyze such conditions as long-term, steady-state seepage conditions, using drained strengths and the highest probable position of the piezometric surface within the slope. For submerged and partially submerged slopes, the possibility of low water events and rapid drawdown should be considered.

(2) Riverbanks are subject to fluctuations in water level, and consideration of rapid drawdown is therefore of prime importance. In many cases, river bank slopes are marginally stable as a result of bank seepage, drawdown, or river current erosion removing or undercutting the toe of the slope.

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Federal Highway Administration

Office of Engineering
Office of Technology Applications
400 Seventh Street, SW
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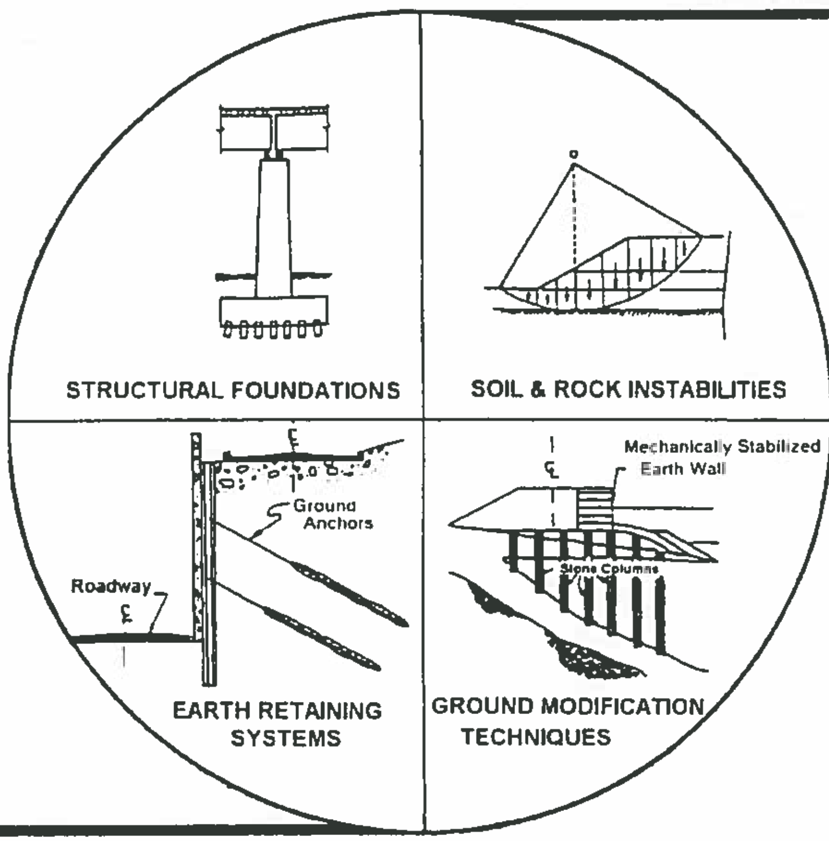
Geotechnical Engineering Circular No. 3

Design Guidance: Geotechnical Earthquake Engineering For Highways

Volume I - Design Principles



4.8 SUMMARY AND CONCLUSIONS

Construction of a new highway required regrading of a hillside. The highest cut slope rose about 44 m above grade at an inclination of 1.5H:1V. The site geology is characterized as sedimentary bedrock of marine origin. It is described as a light tan sandy siltstone with local interbedded clay seams. The bedding dips out at the proposed cut slope at an apparent angle of 12 degrees. The site is located about 7.5 km from a strike-slip fault capable of generating earthquakes of moment magnitude (M_w) of 7.0. A seismic hazard analysis was conducted using five attenuation relationships, resulting in a maximum horizontal ground acceleration of 0.4 g for use in design at the site.

Slope stability criteria called for a static factor of safety of 1.5 and a pseudo-static factor of safety of 1.1 for a seismic coefficient of 0.2 (one-half the peak ground acceleration). Initial slope stability analyses indicated that the proposed grading design would not meet the static and pseudo-static criteria.

A remedial solution was developed to stabilize the slope. The solution was based on using permanent ground anchors located on benches. The ground anchors were also designed for a seismic coefficient of 0.2 g.

"Stability and Performance of Slopes and Embankments – II", R. B. Sood and R. W. Boulanger (eds.), ASCE, 1992

FS=1.5: IS IT APPROPRIATE FOR EMBANKMENT DESIGN?

by Scott A. Ashford,[1] AM, Lawrence H. Roth,[2] F, Sandra L. Madsen,[3] AM, and Donald G. Anderson,[4] M. ASCE

ABSTRACT

This case history discusses an issue that practicing engineers face on all embankment design projects: What is the appropriate minimum factor of safety (FS)? An historical perspective shows that over a period of 55 years, FS=1.5 is a paradigm, evolving from a target value, to a codified value, and finally, to a mandatory minimum expected by review agencies and regulators. However, the literature indicates that FS should vary with the quality and quantity of data, the technical approach, and judgment. The case history illustrates these points, concluding that FS=1.5 is a *good place to start*, but codification limits the designers' ability to exercise geotechnical engineering judgment.

INTRODUCTION

This paper presents a case history of embankment design illustrating an issue that practicing engineers face on all projects involving slope stability: what is the appropriate minimum factor of safety (FS)? Or more importantly, is there a single, acceptable minimum value? FS=1.5 has been practically "codified" as the generally accepted minimum allowable FS for embankment design; on public works projects, proving FS ≥ 1.5 is almost mandatory, regardless of the sophistication or thoroughness of the technical approach. Selecting this value as a minimum acceptable, or "target," FS at the outset

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may make an otherwise stable slope appear "unsafe," depending on the quantity and quality of the data, and the analytical procedures followed.

Unfortunately, a codified value for FS cannot adequately account for the unknowns in the analyses (e.g., variations in shear strength and stratigraphy, pore-water pressures, type and accuracy of the analytical technique). It should be the responsibility of the design engineer to select a minimum acceptable FS based on site conditions, adequacy of the data, and analytical procedures chosen for embankment design. Review agencies, and in some cases regulators, must be afforded the opportunity to review and comment on the target FS in light of how they judge the appropriateness and completeness of the approach. A codified FS, however, reduces or eliminates the role of judgment for the designer, reviewer, and regulator.

The first part of this paper presents an historical perspective on the development of 1.5 as the generally accepted minimum allowable FS for embankment design in the U.S. Following this, a case history illustrates the difficulties and frustrations of trying to meet a codified minimum FS. In this case, a rigorous approach was used to design a large highway embankment on a relatively soft, compressible foundation, and to enable comparison of a calculated FS with a target FS established by a review agency. A less rigorous and less costly approach, however, could have also been used to meet the same target FS, but with greater uncertainty. Following the case history, conclusions are made regarding the question, FS=1.5: Is it appropriate for embankment design?

HISTORICAL PERSPECTIVE

Evolution of FS=1.5. There are surprisingly few citings of a minimum acceptable FS in the literature. Yet, over the years, FS=1.5 has evolved into a paradigm as illustrated in this historical perspective.

At the First International Conference on Soil Mechanics and Foundation Engineering, Terzaghi (1936) discussed the factor of safety for embankment stability, but his brief paper makes no mention of a minimum acceptable FS. Similarly, no other authors at the conference found it necessary or appropriate to define a minimum acceptable value. Seven years later, Terzaghi (1943) stated in *Theoretical Soil Mechanics*, that for stability computations for the $\phi=0$ condition, "The slope angle of the sides of the cut should be so selected that the factor of safety with respect to sliding is equal to 1.5." This represents the first significant citing of a minimum acceptable value.

Five years later, Taylor (1948), in his text, *Fundamentals of Soil Mechanics*, does not define a minimum acceptable FS for embankment stability. In an example embankment stability calculation, the calculated FS is 1.37. Taylor writes that though this value appears low, "It is a typical value, however, and many embankments, which according to engineering experience are safe, have safety factors smaller than this value." Taylor also states "there is no such thing as *the* factor of safety and that when *a* factor of safety is used its meaning should be clearly defined."

In the same year, another text, *Soil Mechanics in Engineering Practice* (Terzaghi and Peck, 1948), expands on Terzaghi's earlier writings and proposes a minimum acceptable FS for embankment stability: "Under normal circumstances, the foundation conditions are not considered satisfactory unless the factor of safety with respect to base failure [of an embankment] during or immediately after construction is at least equal to 1.5." For earth dams, the text states, "The theoretical factor of safety with respect to slope failures should never be less than 1.3 and should preferably be 1.5." The authors go on to say, "Since our knowledge of the conditions for the stability of clay embankments is still incomplete, ..., a theoretical factor of safety of 1.5 should be regarded as the minimum requirement."

Fifteen years later Sherard et al (1963), writing in *Earth and Earth-Rock Dams*, postulate a minimum acceptable FS. The authors write, "At the present state of our ability ..., it is not possible or reasonable to establish any hard and fast criteria for a minimum tolerable safety factor." They go on to discuss the difficulties in doing so, including the various methods available for determining shear strength and the different types of analytical procedures used by practicing engineers. Following this, the authors state, "In spite of these differences, a safety factor of 1.5 has been adopted as a minimum tolerable value for the full reservoir condition on the great majority of projects. The probable end result of this evolutionary process, assuming that no failures occur to alter present thinking, will be that a safety factor of 1.5 will be considered satisfactory when computed with a method of calculation in which side forces are considered ... and with pore-pressures estimated from a steady-state flow net."

Perhaps this evolutionary process was nearing completion when Duncan and Buchignani (1975) gave minimum acceptable values of FS varying from 1.25 to 2.0 as "guidance." They suggest using FS=1.5 when "cost of repair [is] much greater than cost of construction, or [there is] danger to human life or other valuable property if the slope fails" and when the "uncertainty of strength measurements" is small.

Codification. Nearly 40 years passed from the First International Conference to Duncan and Buchignani's manual, and it seems 1.5 evolved into a *good place to start* for the minimum acceptable FS for embankment design. Of importance, all authors referenced above stressed that judgment needed to be used in conjunction with their recommendations. One could assume for a given project, selecting 1.5 or, for that matter, any value as the minimum acceptable FS would be dependent on the quality and amount of subsurface information, the understanding of the groundwater and strength conditions, and the rigorousness of the analytical procedure.

What helps move 1.5 from being *a good place to start* to being a codified value is its adoption by many governmental agencies and regulatory bodies as *the minimum acceptable FS*. For example, in the *Materials Manual* by the California Department of Transportation (Caltrans, 1973), 1.5 is given as the minimum acceptable FS. In the U.S. Navy's *DM-7.01* (USN, 1986), the value of 1.5 is a *mandatory* minimum acceptable FS. California Integrated Waste Management Board's *Closure/Postclosure Regulations* (1990), state that "The [foundation] report must indicate a factor of safety for the critical [landfill] slope of at least 1.5 under dynamic conditions." Absent from these "codes" are instructions for the appropriate procedures to analyze slope stability and determine FS. With many options available for analysis, a specified target minimum FS should also be accompanied by detailed descriptions on the methods to calculate it, as noted by Taylor (1948).

It is likely that these *codified* values were intended to be somewhat flexible, but the fact that they are in print truly limits flexibility for all parties in the design process including the geotechnical engineer, the reviewing agency, and the regulator. Recognize too that little guidance is provided regarding how and when engineering judgment may be used to select a more appropriate minimum FS.

Factors Affecting FS. At the first Stability and Performance of Slopes and Embankments conference, held at Berkeley in 1966, Lowe (1966) presented a state-of-the-art paper entitled "Stability Analysis of Embankments." In it, Lowe notes three aspects that influence uncertainty in stability analyses: analysis of forces, selection of shear strengths, and application of the first two to specific loading conditions. Lowe states that the purpose of any minimum target FS is to account for uncertainty in the analyses, shear strength being the primary source of uncertainty.

The ordinary method of slices, which satisfies only overall moment equilibrium, is a common procedure that both Lowe (1966), and Duncan and Buchignani (1975) say is adequate in most circumstances. However, this method generally gives a lower FS than other more rigorous methods which satisfy all conditions of equilibrium (Wright et al, 1973), such as the method proposed by Janbu (1957). It seems logical, then, that the procedure selected for analysis of forces should therefore influence the value selected for the target minimum FS.

Lowe uses the unconsolidated-undrained (UU) strength for the end-of-construction case for embankment design. However, Ladd, in his Terzaghi Lecture (1991), states that the consolidated-undrained (CU) strength is a better representation of field conditions in this case. Furthermore, Mayne (1988) and Ladd (1991) indicate that there is much more variability in the results of UU triaxial than there is in CU triaxial tests. This suggests that the strengths used in the analysis should also influence the value selected for the target minimum FS.

Lowe (1966) states that, as a "cardinal rule," the tests for shear strength should duplicate field conditions. In the case of a foundation failure of an embankment, Ladd (1991) suggests the direct simple shear (DSS) strength best represents the average field conditions. However, UU and isotropically consolidated undrained (CIU) triaxial compression tests are routinely performed to evaluate shear strength for slope stability because these tests are less costly and are easier to run. These tests often give higher values of shear strength than DSS (Mayne, 1988). It appears that test methods should also influence the value selected for the target minimum FS.

There are many other factors that should be taken into account in selecting a minimum acceptable FS. For example, Duncan and Buchignani (1975) indicate that selection of FS should at least partially depend on the following factors: the uncertainty in slope geometry, the cost of modifying the slope, the cost of the consequences of failure, and whether the slope is temporary or permanent. Clearly these factors cannot be accounted for in a single codified value.

Alternatives. One alternative to a deterministic approach with a codified FS is a limit state approach; for example, the Det norske Veritas' code for design of offshore structures (DnV, 1981) and the *Canadian Foundation Engineering Manual* (1985). In the DnV approach, safety is still verified by determining that the design load will not exceed the design resistance. However, the design load is calculated by multiplying a characteristic load by a load coefficient found in the code. The design resistance is calculated by dividing characteristic strength by material coefficients, also found in the code.

There are two notable conditions in the DnV approach. First, conservative values must be selected for characteristic strength; the code states that greater conservativism must be used when there is much scatter in the raw data, or when limited data are available. Second, if effective stress analyses are performed, the code stipulates that soil strength must be determined using laboratory shear strengths with pore-pressure measurements. On the other hand, if total stress analyses are made, the code requires using soil strengths based on shear strength tests that match field stress conditions as closely as possible. Additionally, the code recommends that test results be interpreted using stress paths. This code provides guidance on the analytical procedures required by the reviewing agency, yet considerable room is left for the use of engineering judgment.

Another alternative is a probabilistic approach, also recommended by DnV, which states that target probability levels should be selected on a case-by-case basis (1981). Reliability and risk analysis in geotechnical engineering was a frequent topic of discussion in journals throughout the 1970s and early 1980s. Whitman's 1981 Terzaghi Lecture, "Evaluating Calculated Risk in Geotechnical Engineering," gives a comprehensive look at the role of reliability in our field (Whitman, 1984). Whitman describes reliability analysis as a way to balance uncertainty with safety while allowing the engineer to exercise judgment more clearly. Whitman explains that reliability analysis is particularly well suited to slope stability and discusses FS=1.5. Because various methods are used by different engineers for stability analyses, "... one slope with a reported factor of safety of 1.5 may actually have little margin of safety, while another with the same reported factor of safety may be virtually proof against failure." Whitman also concludes that "(the) use of FS=1.5 for all slope stability problems implies wide differences in reliability."

Whitman (1984) explains that assignment of numerical value for risk is essential to reliability analysis, and goes on to say: "...there is a real danger that criteria for allowable risk might become fixed and inflexible, thus demanding a precise evaluation of risk beyond what can realistically be achieved. The ... danger is real, and when it happens, important discussions about the objectives and priorities of society are reduced to unprofessional squabbles over the details of an analysis." Similarly, it appears codifying FS establishes a fixed and inflexible criterion which limits the designer's ability to exercise engineering judgment as illustrated in the following case history.

CASE HISTORY

Background. The Great America Parkway Interchange and SR-237 Realignment Project is a large highway project requiring considerable geotechnical design. The project includes elevating 2 miles of existing highway on a 1-million-cubic-yard embankment fill up to 40 feet high on the southern margin of San Francisco Bay, just north of San Jose, California. Several bridge structures will cross existing city streets, a railroad, and creeks, necessitating 10 bridge abutments in the embankment fill. Proposed embankment side slopes are 2H:1V (horizontal to vertical) and abutment slopes are 1.5H:1V. Consolidation settlement is a concern for the bridge structures, and most abutments will require preloading and surcharge prior to bridge construction, resulting in slopes 45 feet high during construction. The fill is unprecedented in the area, and a rigorous geotechnical exploration and laboratory testing program was undertaken.

Exploration and Testing. Nearly 6,000 linear feet of drilling and sampling and 5,000 feet of cone penetration testing were performed to help define subsurface conditions. In addition, the geological history of the Bay margin was reviewed, and the performance of other, smaller embankments in the area were evaluated to better characterize expectations of behavior. Several types of strength tests were performed as well as index and consolidation testing of recovered soil samples. Strength tests included field vane; UU and CIU triaxial compression; and direct simple shear (DSS).

Soil Conditions. Based on the exploration, the site consists primarily of layers of lean clay with varying degrees of overconsolidation caused by alternating sea level stands, desiccation, and groundwater overdraft. There is a relatively thin surface layer of desiccated San Francisco Bay mud, underlain by several layers of Holocene- and Pleistocene-age alluvial clay, and Pleistocene-age Bay clay. A soil profile for the upper 50 feet of the site is presented in Figure 1. Average test results for the soil units shown on the profile are presented in Table 1. Since the embankment fill was to be imported, the designers could specify desirable material properties. Reasonable values for these properties were assumed for the analysis as shown in Table 1.

Stability Analyses. Although the project was funded by a local sales tax levy, Caltrans will be the owner of the constructed highway. This put Caltrans in the role of the review agency, and it stipulated that the factor of safety for the embankment fills meet two requirements. First, during construction, the minimum acceptable calculated FS was to be 1.25 for slopes where "failure would not be catastrophic with respect to impedance of traffic or dangerous to human life or contiguous structures" (Caltrans, 1973). Second, when the freeway was open to traffic, the minimum acceptable FS was to be 1.5, for slopes where "failure could endanger human life, present a serious traffic hazard, or damage costly contiguous structures" (Caltrans, 1973).

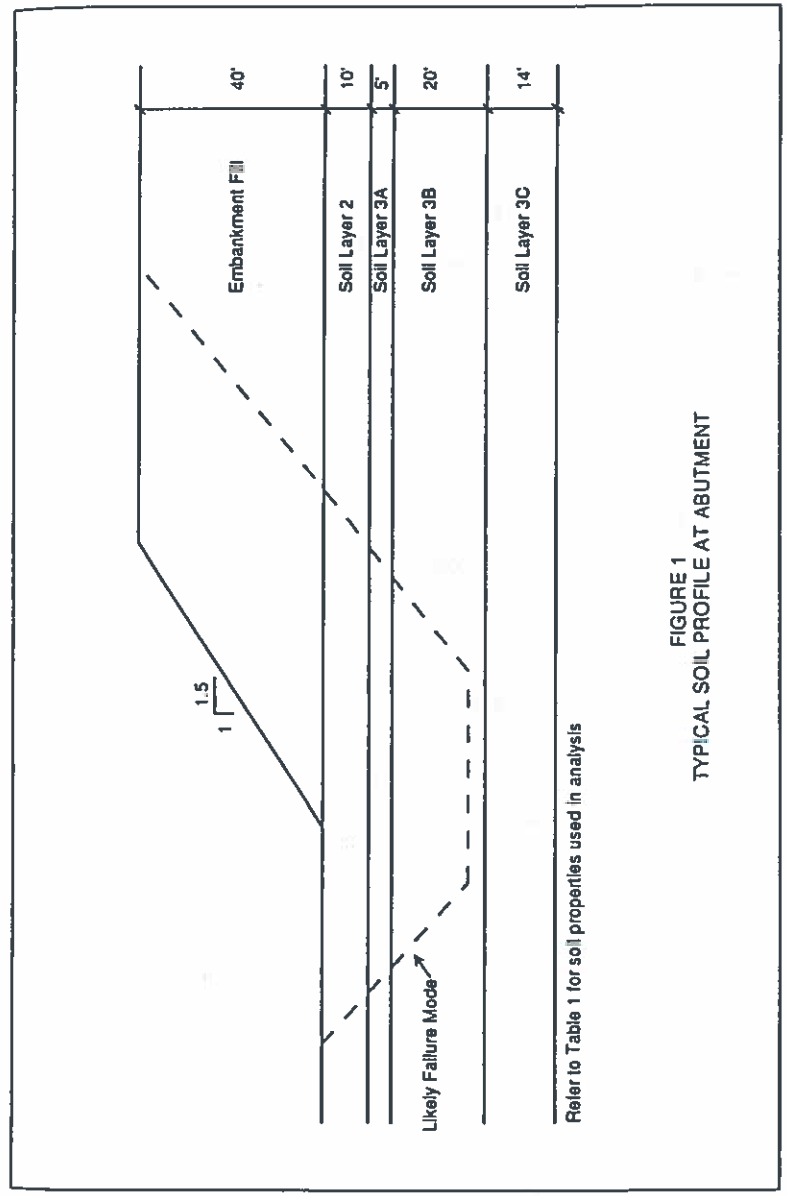

FIGURE 1
TYPICAL SOIL PROFILE AT ABUTMENT

Table 1
Soil Properties

Soil Layer	Saturated Unit Weight (pcf)	Natural Water Content (%)	LL[1]	PI[1]	Design Shear Strength (psf) Before Consolidation DSS	CIU	UU	After Consolidation DSS	CIU
2	118	34	49	26	920	1,530	1,450	1,500	2,560
3A	120	33	34	21	1,150	1,490	1,880	1,590	2,065
3B	120	32	42	19	920	1,220	740	1,110	2,265
3C	130	22	42	23	1,800	2,900	2,480	2,450	3,435

Embankment fill has saturated unit weight = 125 pcf, and friction angle = 35 degrees.
[1]LL=Liquid Limit, PI=Plasticity Index

The stability of embankment slopes was determined using undrained strengths and analysis methods similar to those suggested by Ladd (1991). DSS strengths were selected to model the foundation soil since the likely mode of failure was through a weak foundation layer. Soil conditions were relatively uniform across the site, and the results of the laboratory testing indicated that the soil exhibited normalized properties. The SHANSEP procedure was used to define shear strength of the foundation soils (Ladd and Foott, 1974). DSS correlations with CIU tests were developed in the laboratory program and verified using published correlations (Mayne, 1988) because extensive DSS testing was economically prohibitive.

The computer program STABL (Siegel, 1975) was used to calculate the FS for abutment slopes using Janbu's method of slices (Janbu, 1957). The FS was slightly less than 1.5 for some abutment slopes when SHANSEP and DSS strengths were used. Table 2 shows the results of stability analyses for a typical abutment using DSS strengths.

Table 2
Stability Analysis Results[a]

Strength Basis	Factor of Safety Before Consolidation	After Consolidation
DSS	1.2	1.4
CIU	1.6	2.1
UU	1.3	--

[a]Based on Janbu

The designers were satisfied with FS less than 1.5 based on the quantity and quality of soil strength data and on the ability of the analyses to model anticipated field conditions. However, the designers faced a dilemma trying to meet a codified value for FS using procedures that were not contemplated at the time the codified value was established. In their judgment, the abutments were safe and, using a poker analogy, trying to achieve FS=1.5 was like continuing to draw while holding four aces.

The designers could have used analytical procedures involving greater uncertainty, or that were less representative of field conditions to obtain FS=1.5. For example, Table 2 also shows the results of stability analyses for the same abutment using both CIU and UU strengths. Even though the FS resulting from CIU strengths exceeded 1.5, the slope would not have been any "safer."

To achieve FS = 1.5, the designers adjusted surcharge heights and varied preload schedules to gain strength from consolidation. In some cases, strength gains from consolidation were insufficient to meet FS = 1.5. In these cases, the designers slightly increased the DSS strengths based on observed trends in CIU and UU strength data.

DISCUSSION

Two elements of the analyses merit discussion: shear strength and factor of safety. In Table 1, there is considerable variation between the shear strength for each layer depending on which type of laboratory test is selected. Note that there is no clear correlation between the UU-based strengths and the others. In addition, there is a much wider data spread for the UU tests than for the CIU and DSS tests. This agrees with conclusions made by Mayne (1988) and Ladd (1991) that there is greater uncertainty in the results of UU tests. Based on this alone, it seems inappropriate to require the same minimum acceptable FS for analyses based on UU tests versus those based on CIU or DSS tests.

The analyses needed to demonstrate FS=1.5 when the highway would be open for traffic. To achieve this, the simplest procedure was to first check FS without accounting for strength gains due to consolidation. If adequate (i.e., FS≥1.5), then no further analyses were necessary. If not, then more detailed analyses accounting for strength gains due to consolidation were required. Using CIU strengths from Table 1 as the basis for analysis, the embankment was "safe" after the first check. In contrast, the initial DSS-based analyses did not meet FS=1.5 even accounting for strength gains from complete consolidation. The results for the UU-based analysis fell in between. In the end, FS=1.5 was met by accounting for strength gains because of consolidation under surcharge, and by modifying design shear strengths based on trends observed in all strength data.

Although analyses were made for the same soil, the calculated factors of safety are not equivalent. Each includes various amounts of uncertainty, which should be assessed when a target FS is selected. Obviously, all three analyses should not be required to meet the same FS; yet, each must if the minimum acceptable FS is codified. In this case, a DSS-based analyses with FS=1.4 was, in the designers' judgment, acceptable with regard to public safety. Unfortunately, additional analyses were necessary to satisfy the codified value of FS=1.5.

CONCLUSIONS

The historical perspective provided in the literature illustrates the evolutionary process that has resulted in 1.5 as the generally accepted minimum allowable FS. However, a codified value fails to account for factors requiring consideration before selecting a minimum acceptable FS for a project. Furthermore, using a "minimum factor of safety" can create an uncomfortable dilemma for designers, and the case history shows again what we have known before: no single factor of safety exists.

The factor of safety has often been called the "factor of ignorance" because our ignorance of the true behavior of an embankment and its foundation demands a factor of safety in the first place. Codifying a minimum acceptable FS does not allow for varying degrees of ignorance, or conversely, for using increased reliance on judgment based on greater knowledge. The limit state and probabilistic approaches are able to account for varying degrees of ignorance. Unfortunately, these approaches suffer the same potential downfall if their requirements also become fixed in code. Codifying FS does not reduce ignorance. In fact, it may unwittingly contribute to ignorance by reducing or removing the ability to exercise engineering judgment in slope stability analyses for embankment design.

The question remains, "FS=1.5: Is it appropriate for embankment design?" The answer appears to be yes... and no. Many embankments have been designed and constructed successfully using FS=1.5. Because of this, engineering judgment suggests that this is a *good place to start*. However, as Whitman (1984) points out, some slopes with FS=1.5 are likely far safer than others. And, as presented in this paper, the factor of safety is highly dependent on the method of testing, selection of strength, and analysis of forces. Factors of safety of less than 1.5 should be acceptable on projects where uncertainty is low, and values greater than 1.5 should be required where uncertainty is high. Designers, in concert with review agencies and regulators, should exercise their engineering judgment to select a target FS compatible with the acknowledged limitations of the data and the analytical models. Future investigators should help characterize a target FS by providing additional insight on the sources of uncertainty, and on how to account for uncertainty in selecting a target FS. Judgment says that it should be occasionally greater, occasionally smaller; therefore, FS=1.5 is not always the appropriate minimum acceptable factor of safety for embankment design.

REFERENCES

California Integrated Waste Management Board (1990). *Closure/Postclosure Regulations*, Title 14, California Code of Regulations, Division 7, Chapter 3, Article 7.8, 19-21.

California Department of Transportation (1973). *Materials Manual Volume VI: Foundation Exploration, Testing and Analysis Procedure.*

Canadian Geotechnical Society (1985). *Canadian Foundation Engineering Manual*, Bitech Publishers, Vancouver, B.C.

Det norske Veritas (1981). *Rules for the Construction and Inspection of Offshore Structures*, 8-60.

Duncan, J. M., and Buchignani, A. L. (1975). *An Engineering Manual for Slope Stability Studies*, Geotechnical Engineering Report, Department of Civil Engineering, University of California at Berkeley.

Janbu, N. (1957). "Earth Pressure and Bearing Capacity Calculations by Generalized Procedure of Slices," *Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, London, England, 207-212.

Ladd, C. C. (1991). "Stability Evaluation During Staged Construction," *Journal of Geotechnical Engineering*, ASCE, 17(4), 540-615.

Ladd, C. C. and Foott, R. (1974). "New Design Procedure for the Stability of Soft Clays," *Journal of the Geotechnical Engineering Division*, ASCE, 100(7), 763-784.

Lowe, J., "Stability Analysis of Embankments" (1966). *Proceedings of the Stability and Performance of Slopes and Embankments Conference*, Soil Mechanics and Foundations Division, ASCE, 1-33.

Mayne, P. W. (1988). "Determining OCR in Clays from Laboratory Strength," *Journal of Geotechnical Engineering*, ASCE, 114(1), 76-92.

Sherard, J. L., Woodward, R. J., Giziensky, S. F., and Clevenger, W. A. (1963). *Earth and Earth-Rock Dams*, John Wiley and Sons, New York, N.Y.

Siegel, R. A. (1975). *Computer Analysis of General Slope Stability Problems*, Technical Report No. 75-8, Joint Highway Research Project, Purdue University, West Lafayette, Indiana.

Taylor, D. W. (1948). *Fundamentals of Soil Mechanics*, John Wiley and Sons, New York, N.Y.

Terzaghi, K. (1936). "Critical Height and Factor of Safety of Slopes Against Sliding," *Proceedings of the First International Conference on Soil Mechanics*, Cambridge, Mass., Vol. 1, 156-161.

Terzaghi, K. (1943). *Theoretical Soil Mechanics*, John Wiley and Sons, New York.

Terzaghi, K. and Peck, R. B. (1948). *Soil Mechanics in Engineering Practice*, John Wiley and Sons, New York, N.Y.

United States Navy (1986). *Design Manual 7.01; Soil Mechanics*, Naval Facilities Engineering Command, Alexandria, Virginia.

Whitman, R. V. (1984) "Evaluating Calculated Risk in Geotechnical Engineering," *Journal of the Geotechnical Engineering Division*, ASCE, 110(2), 145-216.

Wright, S. G., Kulhawy, F. H., and Duncan, J. M. (1973). "Accuracy of Equilibrium Slope Analysis," *Journal of the Soil Mechanics and Foundations Division*, ASCE, 99(SM10), 783-791.

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Soil Mechanics

DESIGN MANUAL 7.01
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CHAPTER 7. SLOPE STABILITY AND PROTECTION

Section 1. INTRODUCTION

1. SCOPE. This chapter presents methods of analyzing stability of natural slopes and safety of embankments. Diagrams are included for stability analysis, and procedures for slope stabilization are discussed.

2. APPLICATIONS. Overstressing of a slope, or reduction in shear strength of the soil may cause rapid or progressive displacements. The stability of slopes may be evaluated by comparison of the forces resisting failure with those tending to cause rupture along the assumed slip surface. The ratio of these forces is the factor of safety.

3. RELATED CRITERIA. Excavations, Earth Pressures, Special Problems - See DM-7.2, Chapters 1, 2 and 3 and DM-7.3, Chapter 3.

4. REFERENCE. For detailed treatment on subject see Reference 1, Landslide Analyses and Control, by the Transportation Research Board.

Section 2. TYPES OF FAILURES

1. MODES OF SLOPE FAILURE. Principal modes of failure in soil or rock are (i) rotation on a curved slip surface approximated by a circular arc, (ii) translation on a planar surface whose length is large compared to depth below ground, and (iii) displacement of a wedge-shaped mass along one or more planes of weakness. Other modes of failure include toppling of rockslopes, falls, block slides, lateral spreading, earth and mud flow in clayey and silty soils, and debris flows in coarse-grained soils. Tables 1 and 2 show examples of potential slope failure problems in both natural and man-made slopes.

2. CAUSES OF SLOPE FAILURE. Slope failures occur when the rupturing force exceeds resisting force.

a. Natural Slopes. Imbalance of forces may be caused by one or more of the following factors:

(1) A change in slope profile that adds driving weight at the top or decreases resisting force at the base. Examples include steepening of the slope or undercutting of the toe.

(2) An increase of groundwater pressure, resulting in a decrease of frictional resistance in cohesionless soil or swell in cohesive material. Groundwater pressures may increase through the saturation of a slope from rainfall or snowmelt, seepage from an artificial source, or rise of the water table.

TABLE 1

Analysis of Stability of Natural Slopes

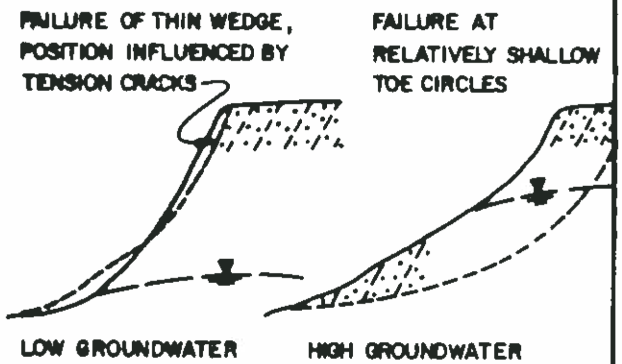
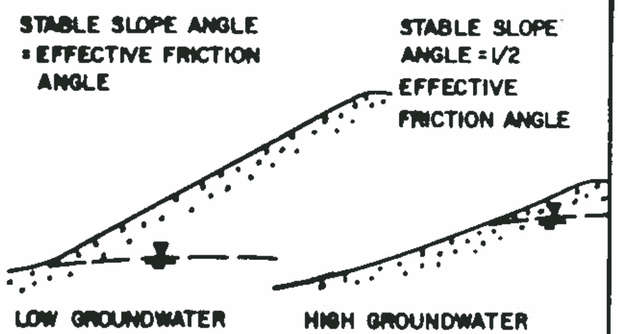
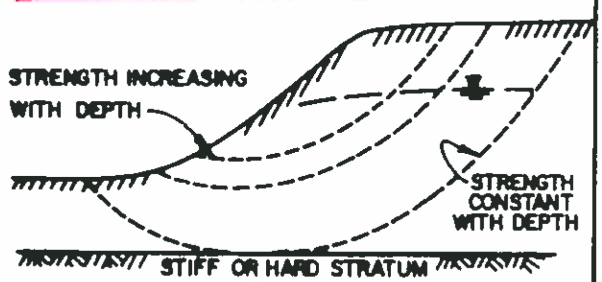
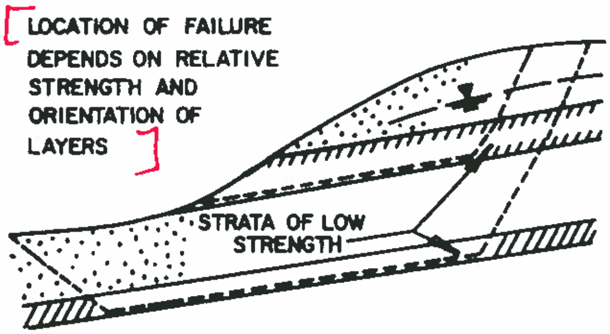
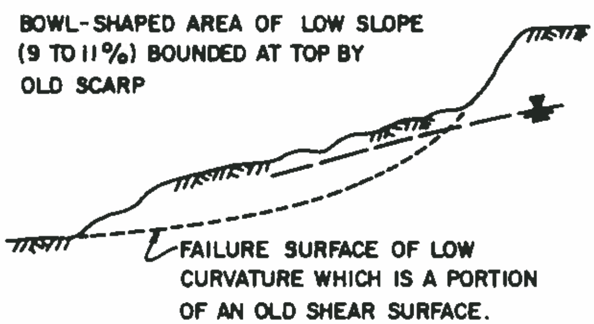
 (1) SLOPE IN COARSE-GRAINED SOIL WITH SOME COHESION	WITH LOW GROUNDWATER, FAILURE OCCURS ON SHALLOW, STRAIGHT, OR SLIGHTLY CURVED SURFACE. PRESENCE OF A TENSION CRACK AT THE TOP OF THE SLOPE INFLUENCES FAILURE LOCATION. WITH HIGH GROUNDWATER, FAILURE OCCURS ON THE RELATIVELY SHALLOW TOE CIRCLE WHOSE POSITION IS DETERMINED PRIMARILY BY GROUND ELEVATION. ANALYZE WITH EFFECTIVE STRESSES USING STRENGTHS c' AND ϕ' FROM CD TESTS. PORE PRESSURE IS GOVERNED BY SEEPAGE CONDITION. INTERNAL PORE PRESSURES AND EXTERNAL WATER PRESSURES MUST BE INCLUDED.
 (2) SLOPE IN COARSE-GRAINED, COHESIONLESS SOIL	STABILITY DEPENDS PRIMARILY ON GROUND-WATER CONDITIONS. WITH LOW GROUNDWATER, FAILURES OCCUR AS SURFACE SLOUGHING UNTIL SLOPE ANGLE FLATTENS TO FRICTION ANGLE. WITH HIGH GROUNDWATER, STABLE SLOPE IS APPROXIMATELY 1/2 FRICTION ANGLE. ANALYZE WITH EFFECTIVE STRESSES USING STRENGTH ϕ'. SLIGHT COHESION APPEARING IN TEST ENVELOPE IS IGNORED. SPECIAL CONSIDERATION MUST BE GIVEN TO POSSIBLE FLOW SLIDES IN LOOSE, SATURATED FINE SANDS.
LOCATION OF FAILURE DEPENDS ON VARIATION OF SHEAR STRENGTH WITH DEPTH  (3) SLOPE IN NORMALLY CONSOLIDATED OR SLIGHTLY PRECONSOLIDATED CLAY	FAILURE OCCURS ON CIRCULAR ARCS WHOSE POSITION IS GOVERNED BY THEORY, SEE FIG. 3. POSITION OF GROUNDWATER TABLE DOES NOT INFLUENCE STABILITY UNLESS ITS FLUCTUATION CHANGES STRENGTH OF THE CLAY OR ACTS IN TENSION CRACKS. ANALYZE WITH TOTAL STRESSES, ZONING CROSS SECTION FOR DIFFERENT VALUES OF SHEAR STRENGTHS. DETERMINE SHEAR STRENGTH FROM UNCONFINED COMPRESSION TEST, UNCONSOLIDATED UNDRAINED TRIAXIAL TEST OR VANE SHEAR.

TABLE 1 (continued)

Analysis of Stability of Natural Slopes

 (4) SLOPE IN STRATIFIED SOIL PROFILE	LOCATION OF FAILURE PLANE IS CONTROLLED BY RELATIVE STRENGTH AND ORIENTATION OF STRATA. FAILURE SURFACE IS COMBINATION OF ACTIVE AND PASSIVE WEDGES WITH CENTRAL SLIDING BLOCK CHOSEN TO CONFORM TO STRATIFICATION. ANALYZE WITH EFFECTIVE STRESS USING c' AND ϕ' FOR FINE-GRAINED STRATA AND ϕ' FOR COHESIONLESS MATERIAL.
 (5) DEPTH CREEP MOVEMENTS IN OLD SLIDE MASS	STRENGTH OF OLD SLIDE MASS DECREASES WITH MAGNITUDE OF MOVEMENT THAT HAS OCCURRED PREVIOUSLY. MOST DANGEROUS SITUATION IS IN STIFF, OVER-CONSOLIDATED CLAY WHICH IS SOFTENED, FRACTURED, OR SLICKENSIDED IN THE FAILURE ZONE.

(3) Progressive decrease in shear strength of the soil or rock mass caused by weathering, leaching, mineralogical changes, opening and softening of fissures, or continuing gradual shear strain (creep).

(4) Vibrations induced by earthquakes, blasting, or pile-driving. Induced dynamic forces cause densification of loose sand, silt, or loess below the groundwater table or collapse of sensitive clays, causing increased pore pressures. Cyclic stresses induced by earthquakes may cause liquefaction of loose, uniform, saturated sand layers (see DM-7.3, Chapter 1).

b. Embankment (Fill) Slopes. Failure of fill slopes may be caused by one or more of the following factors:

(1) Overstressing of the foundation soil. This may occur in cohesive soils, during or immediately after embankment construction. Usually, the short-term stability of embankments on soft cohesive soils is more critical than the long-term stability, because the foundation soil will gain strength as the pore water pressure dissipates. It may, however, be necessary to check the stability for a number of pore pressure conditions. Usually, the critical failure surface is tangent to the firm layers below the soft subsoils.

(2) Drawdown and Piping. In earth dams, rapid drawdown of the reservoir causes increased effective weight of the embankment soil thus reducing stability. Another potential cause of failure in embankment slopes is subsurface erosion or piping (see Chapter 6 for guidance on prevention of piping).

(3) Dynamic Forces. Vibrations may be induced by earthquakes, blasting, pile driving, etc.

c. Excavation (Cut) Slopes. Failure may result from one or more of the factors described in (a). An additional factor that should be considered for cuts in stiff clays is the release of horizontal stresses during excavation which may cause the formation of fissures. If water enters the fissures, the strength of the clay will decrease progressively. Therefore, the long-term stability of slopes excavated in cohesive soils is normally more critical than the short-term stability. When excavations are open over a long period and water is accessible, there is potential for swelling and loss of strength with time.

3. EFFECT OF SOIL OR ROCK TYPE.

a. Failure Surface. In homogeneous cohesive soils, the critical failure surface usually is deep whereas shallow surface sloughing and sliding is more typical in homogeneous cohesionless soils. In nonhomogeneous soil foundations the shape and location of the failure depends on the strength and stratification of the various soil types.

b. Rock. Slope failures are common in stratified sedimentary rocks, in weathered shales, and in rocks containing platy minerals such as talc, mica, and the serpentine minerals. Failure planes in rock occur along zones of weakness or discontinuities (fissures, joints, faults) and bedding planes (strata). The orientation and strength of the discontinuities are the most

c. Finite Element Method. This method is extensively used in more complex problems of slope stability and where earthquake and vibrations are part of total loading system. This procedure accounts for deformation and is useful where significantly different material properties are encountered.

2. FAILURE CHARACTERISTICS. Table 1 shows some situations that may arise in natural slopes. Table 2 shows situations applicable to man-made slopes. Strength parameters, flow conditions, pore water pressure, failure modes, etc. should be selected as described in Section 4.

3. SLOPE STABILITY CHARTS.

a. Rotational Failure in Cohesive Soils ($\emptyset = 0$)

(1) For slopes in cohesive soils having approximately constant strength with depth use Figure 2 (Reference 4, Stability Analysis of Slopes with Dimensionless Parameters, by Janbu) to determine the factor of safety.

(2) For slope in cohesive soil with more than one soil layer, determine centers of potentially critical circles from Figure 3 (Reference 4). Use the appropriate shear strength of sections of the arc in each stratum. Use the following guide for positioning the circle.

(a) If the lower soil layer is weaker, a circle tangent to the base of the weaker layer will be critical.

(b) If the lower soil layer is stronger, two circles, one tangent to the base of the upper weaker layer and the other tangent to the base of the lower stronger layer, should be investigated.

(3) With surcharge, tension cracks, or submergence of slope, apply corrections of Figure 4 to determine safety factor.

(4) Embankments on Soft Clay. See Figure 5 (Reference 5, The Design of Embankments on Soft Clays, by Jakobsen) for approximate analysis of embankment with stabilizing berms on foundations of constant strength. Determine the probable form of failure from relationship of berm and embankment widths and foundation thickness in top left panel of Figure 5.

4. TRANSLATIONAL FAILURE ANALYSIS. In stratified soils, the failure surface may be controlled by a relatively thin and weak layer. Analyze the stability of the potentially translating mass as shown in Figure 6 by comparing the destabilizing forces of the active pressure wedge with the stabilizing force of the passive wedge at the toe plus the shear strength along the base of the central soil mass. See Figure 7 for an example of translational failure analysis in soil and Figure 8 for an example of translational failure in rock.

Jointed rocks involve multiple planes of weakness. This type of problem cannot be analyzed by two-dimensional cross-sections. See Reference 6, The Practical and Realistic Solution of Rock Slope Stability, by Von Thun.

5. REQUIRED SAFETY FACTORS. The following values should be provided for reasonable assurance of stability:

(1) Safety factor no less than 1.5 for permanent or sustained loading conditions.

(2) For foundations of structures, a safety factor no less than 2.0 is desirable to limit critical movements at foundation edge. See DM-7.2, Chapter 4 for detailed requirements for safety factors in bearing capacity analysis.

(3) For temporary loading conditions or where stability reaches a minimum during construction, safety factors may be reduced to 1.3 or 1.25 if controls are maintained on load application.

(4) For transient loads, such as earthquake, safety factors as low as 1.2 or 1.15 may be tolerated.

6. EARTHQUAKE LOADING. Earthquake effects can be introduced into the analysis by assigning a disturbing force on the sliding mass equal to kW where W is the weight of the sliding mass and k is the seismic coefficient. For the analyses of stability shown in Figure 9a, k_sW is assumed to act parallel to the slope and through the center of mass of the sliding mass. Thus, for a factor of safety of 1.0:

$$Wb + k_sWh = FR$$

The factor of safety under an earthquake loading then becomes

$$F_{Se} = \frac{FR}{Wb + k_sWh}$$

To determine the critical value of the seismic efficient (k_{cs}) which will reduce a given factor of safety for a stable static condition (F_{So}) to a factor of safety of 1.0 with an earthquake loading ($F_{Se} = 1.0$), use

$$k_{cs} = \frac{b}{h}(F_{So} - 1) = (F_{So}-1)\sin\theta$$

If the seismic force is in the horizontal direction and denoting such force as k_{ch} W, then $k_{ch} = (F_{So}-1)\tan\theta$.

For granular, free-draining material with plane sliding surface (Figure 9b): $F_{So} = \tan\phi/\tan\theta$, and $k_{cs} = (F_{So}-1)\sin\theta$.

Based on several numerical experiments reported in Reference 7, Critical Acceleration Versus Static Factor of Safety in Stability Analysis of Earth Dams and Embankments, by Sarma and Bhave, k_{ch} may be conservatively represented as $k_{ch} \approx (F_{So}-1)0.25$.

The downslope movement U may be conservatively predicted based on Reference 8, Effect of Earthquakes on Dams and Embankments, by Newmark as:

$$U = \frac{V^2}{2g\,k_{cs}} \cdot \frac{A}{k_{cs}}$$